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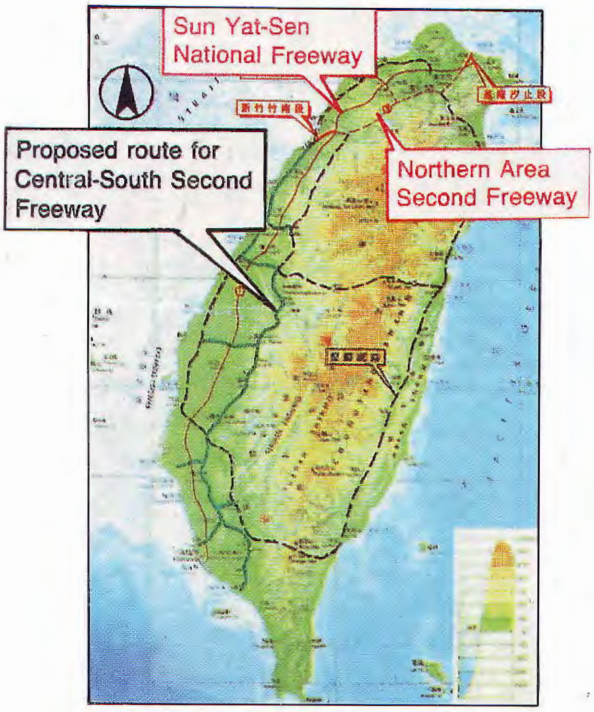

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REAAA Journal

The Journal is a key feature and critical to the on-going success of REAAA as a facilitator of intra-regional technology transfer. Unfortunately, it has proven very difficult to keep the Journal going. While the Technical Committee is endeavouring its best to procure papers, you the members can also play your part in assisting us.

An Editorial Panel comprising of Associate Editors, one from each country on the region will be established soon. They will be responsible for encouraging papers from their own country. Papers from the region on technical matters of interest to other countries in the region or from countries outside the region that have special relevance to the region are sought.

I hope that you will give our Associate Editors your full support, not only to keep the Journal going, but more so that we may be able to fulfill the Journal's objective of facilitating intra-regional technology transfer.

Editor

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COVER: Spiral ramps on Metropolitan Expressway, Japan.

The Katahira Awards

The late Mr. Nobutaka Katahira, a Past President of REAAA generously willed ¥3.0 million to REAAA, which is now kept in fixed deposit known as the Katahira Fund.

The REAAA Governing Council, has decided that the interest accruing from the above Fund be used to finance the Katahira Awards, which will be given to the three best technical papers from *young* authors presented during each REAAA Conference.

The Katahira Awards were given for the first time during the 7th REAAA Conference held in June 1992 in Singapore. The three technical papers which won the awards are:

1. *"Highway Maintenance Planning and Budgetting at Network Level Using a Genetic Algorithm"* by Tan Choon Yong, Dr Fwa Tien Fang and Chan Weng Tat.

2. *"Queue Management and Monitoring in Urban Traffic Control Systems"* by James Y. K. Luk.

3. *"Study on Factors Causing Rutting of Pavement and Design of Surface Course"* by Hiromi Tsurukubo, Tsutomu Nakanishi and Yoshio Takayanagi.

Council has agreed that the three papers above be reprinted in this issue of the Journal to benefit those members who were not at the Conference.

HIGHWAY MAINTENANCE PLANNING & BUDGETING AT NETWORK LEVEL USING A GENETIC ALGORITHM

by

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ABSTRACT

This paper describes the use of a relatively new AI research tool, Genetic Algorithms, to address the problem of optimal budget allocation and maintenance planning over a certain period to maintain pavement network at an acceptable performance level. The system currently being developed, PAVENET, helps to determine when maintenance works should be performed for the different distresses of the various pavement segments of the network over a planning horizon of 20 years. The purpose of such a long period is to ensure that an optimum strategy for the initial years does not lead to crucial repairs being deferred and becoming unacceptably concentrated in a particular year. The economic analysis is based on cost-benefit principles and discounted cash-flow procedures are used. A Genetic Algorithm is adapted for the use in PAVENET to overcome the combinatorial explosion of the optimisation problem.

1. INTRODUCTION

The main problem of a highway maintenance planning and budgeting system at network level is to determine a budget allocation schedule that will maintain a pavement network at an acceptable performance level. Ideally, we would also want to do this at the lowest economic cost whilst maintaining the required performance levels.

Over the past decade, there has been a number of trends and countertrends in the evolution of systems being used to manage highway pavement maintenance planning and budgeting (1). PAVER was one system that was developed to manage more effectively the maintenance of a network of streets and parking lots (2). While PAVER was created to cater to the specific needs of one particular user group to assist management at both the network level and project level over a 5-year planning period, PAVENET was conceived as a general prototype system using AI for network level management over a planning period of 20 years.

2. MAIN FEATURES OF PAVENET

PAVENET is a multi-objective system and its main objectives are:

- ☐ Minimise the present worth of repair costs over the planning horizon.
- ☐ Maximise the usage of allocated budgets.

It has to achieve these objectives subject to the following constraints :

- ☐ Maintain highway conditions at acceptable levels.
- ☐ Budget constraints imposed by the user must be observed.
- ☐ No occurrence of excessive repair needs in the long term.
- ☐ No scheduling of same repairs in consecutive years.

In the prototype version of PAVENET presented here, only the following types of repair and their corresponding PCIs (Pavement Condition Indices) would be considered:

TABLE 1: Type of Repairs

TYPE OF REPAIR	CORRESPONDING PCI
CS (Crack Sealing)	CP (Cracking)
PL (Premix Levelling)	R (Rut Depth)
SC (Seal Coating)	SK (Skid Resistance)

It is important to relate fairly accurately pavement distresses to serviceability and performance (3). Each PCI is represented by an appropriate pavement performance equation. The values of the PCIs are each characterised by a warning level set by the highway engineer. The PCIs are functions of age after construction or repair. Once the warning level of any PCI for any

particular segment is triggered, the segment would be deemed to be due for the appropriate repair. Such a repair is considered crucial and should be done in the year in which its PCI value triggers the warning level. A non-urgent repair is one where excess budget can be utilised to make good some of the existing defects although their warning levels have not been activated yet. Fig. 1 illustrates the relation of a PCI against time.

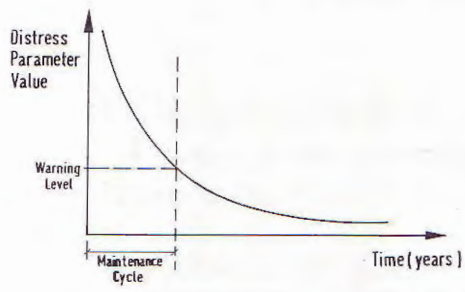

FIGURE 1: Distress Parameter

It has been recognised that there is a need to consider both the short-term and the long-term effects of optimising highway pavement maintenance planning and budgeting. For example, an optimised strategy for the current work-year should not give rise to a situation where the cost of crucial repairs five years later would be ten times the current year's budget. The occurrence of such an unfavourable situation, where the cost of crucial repairs for any year in the planning horizon would definitely exceed the allocated of predetermined budget ceiling, should be prevented and for practical reasons, a 20-year planning horizon is adopted here.

The 20-year period is divided into active and passive planning periods. An active planning period is the length of time where budget allocated by the user is used to determine the crucial as well as non-urgent repairs to be done during the period. In the passive planning periods, only crucial repairs are considered. The main purpose of passive planning periods is to check against the occurrence of excessive repair needs in the long term. Each planning period is taken as a year and in the prototype version, the first two years are active planning years while the remaining years are passive planning years.

3. GENETIC ALGORITHM

Genetic evolution as a computational technique was first proposed and analysed by Holland (5). It has been elaborated and refined by a number of researchers (6,7,8) and has been applied successfully in various domains (9,10).

At the heart of all AI techniques is the question of how best to represent knowledge in a computer and how best to get real-world knowledge into an internal representation. For the GA (Genetic Algorithm), it involves suitably coding the parameter space (in this case, all the non-urgent repairs of all the segments in the network for all active planning years) as an n-bit chromosome. An individual chromosome is called a *genotype*, and the bit values at each position of a genotype are called *alleles*. A cluster of chromosomes is called the *population*. The function value of a genotype is called the genotype's *fitness* or *figure of merit*. The internal representation is illustrated in Fig. 2.

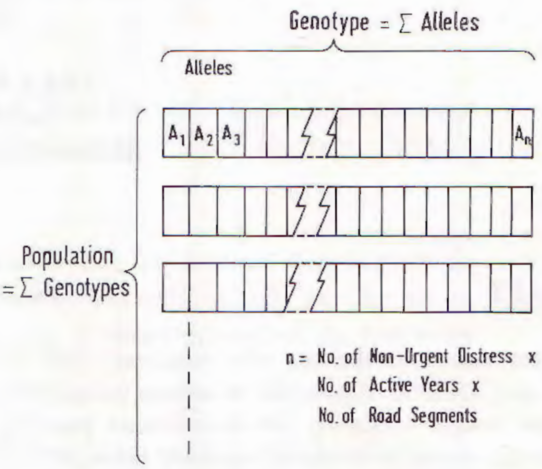

FIGURE 2: Internal Representation

A genotype is grouped by segments corresponding to the network segments. Within each network segment, there are sub-segments corresponding to the active planning years. Finally, within each active planning year are the alleles representing the repairs or otherwise of the corresponding distresses.

A more comprehensive description of the Genetic Algorithm is outside the scope of this paper, and it will not be further discussed here.

To control excessive repair needs within the planning horizon, budget ceilings are set each workyear. The budget ceilings for the passive planning years are determined by the user. They are assumed to be increasing at a rate of 6% per annum with respect to the last active planning year budget in the first series of test runs.

Table 2 shows the budget allocation for a "good" solution obtained through the GA. The first two years are active planning years as explained earlier and the remaining are passive planning years. The uneven budget allocation for urgent repairs in the passive planning years is due mainly to:

- **The age distribution of the segments in the network.** When the majority of the segments are of a particular age, it is normal to expect urgent repairs to be concentrated in certain years.
- **The small number of distresses and segments.** The number of distresses and segments is intentionally kept small in this prototype study to test and refine the use

TABLE 2: NUS Pavenet System

COST SUMMARY RESULTS			
YEAR	COST OF URGENT REPAIRS	TOTAL COST	PRESENT WORTH (COST)
1 ST	$ 22 868.98	$ 29 746.45	$ 29 746.45
2 ND	$ 81 908.98	$ 123 668.98	$ 116 668.85
3 RD	$ 59 112.00	$ 59 112.00	$ 52 609.47
4 TH	$ 238 140.00	$ 238 140.00	$ 199 946.94
5 TH	$ 136 980.00	$ 136 980.00	$ 108 500.99
6 TH	$ 119 225.12	$ 119 225.12	$ 89 114.36
7 TH	$ 332 355.12	$ 332 355.12	$ 234 297.26
8 TH	$ 242 180.92	$ 242 180.14	$ 161 064.14
9 TH	$ 141 229.88	$ 141 229.88	$ 88 609.37
10 TH	$ 216 728.98	$ 216 728.98	$ 128 281.55
11 TH	$ 128 880.00	$ 128 880.00	$ 71 965.92
12 TH	$ 99 612.00	$ 99 612.00	$ 52 474.36
13 TH	$ 221 940.00	$ 221 940.00	$ 110 297.38
14 TH	$ 124 223.12	$ 124 223.12	$ 58 240.65
15 TH	$ 193 827.14	$ 193 827.14	$ 85 729.93
16 TH	$ 375 740.92	$ 375 740.92	$ 156 783.56
17 TH	$ 162 397.88	$ 162 397.88	$ 63 927.32
18 TH	$ 102 500.98	$ 102 500.98	$ 38 065.22
19 TH	$ 246 240.00	$ 246 240.00	$ 86 268.66
20 TH	$ 112 680.00	$ 112 680.00	$ 37 242.21
TOTAL	**$ 3 358 802.04**	**$ 3 407 439.51**	**$ 1 969 834.58**

of GA as a PMS. With more distresses and segments, there is a likelihood of the passive years budget allocation having a more gentle profile.

- **The short active planning period.** With a longer active planning period, non-urgent repairs can be phased out to even off the budget requirement in the passive planning years.

The normal age distribution represents the case of a network which saw slow development at the start, followed by a flurry of construction to meet the escalating needs of a rising community and a slowing down when the highway infrastructure is in place. Problems arise when the network ages and maintenance is required. The problems can become acute on a normal inspect-and-repair basis without considering the fact that the majority of the highway pavements will become due for urgent repair at the same time sometime in the future. One of PAVENET's primary aim is not to allow pavement deterioration to surpass the maintenance resources available to retain the network in a stable condition. It is with the intention of monitoring and avoiding this excessive repair needs in the future that passive planning years concept was introduced.

Initial test runs with the allocated budgets just sufficient for urgent repairs in the active planning years resulted in PAVENET reporting that budget ceiling were exceeded in the passive planning years. This clearly indicated the necessity of preventive repairs to avoid excessive repair needs in the future. Unless the network has a uniform age distribution, it is not advisable to repair the highway only when the distresses become critical.

Results from the prototype version of PAVENET are encouraging and subsequent versions will be able to incorporate, among other things, non-constant maintenance cycles and a longer active planning period.

4. TEST RUNS

In general terms, PAVENET will perform the following operations:

(1) Calculate the budget requirements for urgent repair in each workyear of the planning horizon.

(2) Check to see if the budget ceiling is exceeded for any workyear of the planning horizon :

(a) If budget ceiling is exceeded, a warning message is issued and advice given to the user to rectify the situation. Then repeat (2).

(b) If not, proceed to (3).

(3) Create the chromosomal representation and the initial population of genotypes for the GA(Genetic Algorithm).

(4) Use the GA to search for a good solution that meets the system's objectives and satisfies the constraints. The excess budget in the active planning years is used for allocation to non-urgent repairs.

(5) Decode the good solution obtained.

(6) Generate the following summary results :

(a) Performance statistics of the Genetic Algorithm.

(b) Annual budget allocation for the network over the 20-year planning horizon.

(c) The network repair strategy over the 20-year planning horizon.

In the initial model used for the prototype study, a small 180 lane-km network of 20 highway segments is used, with each segment having 3 independent types of repair. The age of the segments in the network is assumed to have a normal distribution as shown in Fig. 3.

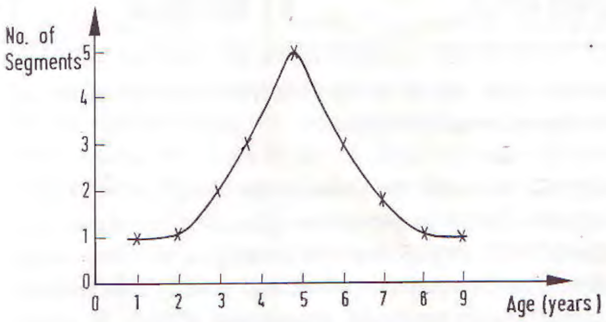

FIGURE 3: Normal Age Distribution

The performance of the prototype was "tuned" over many test runs involving changes to a number of parameters, which affect the behaviour of the GA. Some of the parameters, including those that are scheduled for testing in subsequent versions of PAVENET, are listed below :

- ☐ Rates of mutation (one of the GA operators)
- ☐ Size of the parent pool.
- ☐ Size of the offspring pool.
- ☐ Number of cross-over points for the cross-over operator.
- ☐ Different combinations of GA operators such as cross-over, mutation and inversion.
- ☐ Varying population size that is dependent on the rate of convergence.

5. CONCLUSION

Initial test runs on the prototype version of PAVENET on a small network of highway segments with a normal age distribution have indicated that budget cannot be allocated just for the urgent repairs in the active planning years. This would cause excessive budget requirement in the passive planning period. The following are some of the measures that can be taken to prevent such an unfavourable situation:

- ☐ Allocate budget for non-urgent repairs in the active planning years.
- ☐ Increase the period of the active planning phase. This would allow a more even distribution of non-urgent repairs.

The prototype version has also indicated that the Genetic Algorithm can be a useful tool in PMS. Although the results have been promising, there is still much scope for further development in the use of GA as it is a relatively new AI research tool that has been developed only over the last 20 years. A more in-depth treatment is scheduled for publication in the later part of 1992.

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QUEUE MANAGEMENT AND MONITORING IN URBAN TRAFFIC CONTROL SYSTEMS

by

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ABSTRACT

This paper addresses the issue of queue management in the context of managing travel demand, congestion, and capacity. The role of an urban traffic control system in monitoring and managing queues in oversaturated or gridlock situation is emphasised. A control philosophy of equal saturation to conflicting traffic movements may not be appropriate if the blocking of an intersection due to the downstream queue frequently occurs. A more active approach than adaptive signal control is needed; techniques such as the gating of traffic and the allocation of priorities according to queue storage space should be considered. A first step towards queue management is to introduce on-line queue monitoring in a control system. Empirical studies carried out with the SCATS and TRACS system in Australia have demonstrated the practicality of this concept. The concept is now implemented in TRACs for refinement in Brisbane and other provincial cities.

1. INTRODUCTION

An urban traffic control (UTC) system is an effective tool in coordinating traffic signals to reduce delay, stops, fuel consumption, etc. As congestion levels in major cities increase, UTC systems should be used not just to control the green and red signal timings, but also to implement traffic management strategies. Congestion in many cities have become so acute that any increase in road capacity due to intersection improvements is quickly taken up by increases in travel demand, and travel speed (or delay) remains at the same level as before. Many intersections operate at capacity and there appears to be a huge suppressed demand.

It is well-know that improvements in signal timings will result in more traffic being attracted to an approach that receives a better phase or offset treatment. Research in the existence of this positive feedback and the need for congestion management were reported in Rahmann (1972) and Luk (1978), amongst others. Congestion and environmental issues have to be tackled within a demand and supply framework. McLean (1991) suggested that non-build measures that deal with these issues could be broadly classified as follows (Fig.1):

(a) *Travel demand management* measures – these measures include: high occupancy lanes, car-pooling, road pricing, parking control, park-and-ride facilities, better integration of public transport modes, and land-use planning.

(b) *Congestion management* measures – these measures attempt to minimise the congestion impacts of the prevailing demand level, which is accepted as given; examples include queue and incident detection and management, peak spreading and motorist information systems.

(c) *Capacity management* measures – these measures are concerned with maximising the traffic capacity from an existing road network; examples are linked signal systems, ramp metering, and other traffic engineering techniques such as turn bans and clearways.

Capacity management measures in (c) have been particularly successful in the last two decades in meeting the increased demand of car travel. In Australia they enabled the arterial road system in capital cities to cope with congestion in the absence of comprehensive freeway networks. It now appears that measures in the categories of travel demand and congestion management should receive priority in addressing the issue of congestion.

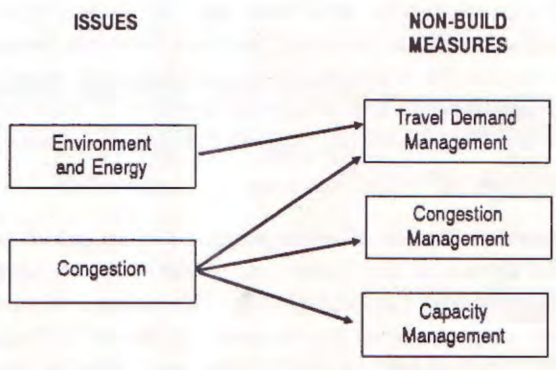

FIGURE 1: Context of Congestion Management in Dealing with Environmental and Congestion Issues

This paper is concerned with the role of a UTC system in the management and monitoring of queue lengths within the context of congestion management described above. Strategies for queue management in oversaturated or 'gridlock' situations are reviewed. Methods for on-line queue monitoring are reported. Empirical studies based on the control systems SCATS (Lowrie 1982; 1988) in Melbourne and TRACS (Lees 1988) in Brisbane are described.

2. QUEUE MANAGEMENT

UTC systems that are fully or partially traffic responsive allocates more green phase times to those approaches that are more congested. This is appropriate as long as queue storage is not an issue and a gridlock situation does not occur. If the downstream queue blocks an upstream intersection, measures other than the 'equal saturation' approach are required.

With SCATS, the user decides which route should be favoured and where the queues can be tolerated. As cycle length increases, the phase times increase as a percentage of the cycle length. An option is available which inhibits this increase at a specified cycle length, provided the degree of saturation exceeds a certain level. The additional cycle length is then dedicated to the phase specified by the traffic engineer. This provides a maximum through-put mode of operation that is capable of increasing the capacity when required and quickly recovering from traffic incidents.

Other measures to minimise blocking effects include:

(a) employing simultaneous offsets rather than travel time offsets,

(b) avoiding the use of long cycle lengths, and

(c) 'gating' or storing traffic of a bottleneck and passing it through the signal so that the downstream congestion is reduced.

There will be other techniques practised by a traffic engineer to handle problems specific to a particular signalised network. However, all these measures basically react to changes in the prevailing traffic demand, hopefully with fast response and acceptable stability. A more active approach than adaptive signal timings is needed for congestion management.

A notable example of strategic queue management with traffic signals is the 'zones and collar' scheme reported in Vincent and Layfield (1977). The scheme attempted to discourage car travel into the inner city area of Nottingham in UK. Traffic signals at selected sites were timed so that car access to radial routes from adjacent residential zones was limited and priority access points were allocated to buses. Further gating of car traffic was introduced at signals along the boundary or collar of the inner city area.

The experimental scheme was discontinued after one year of operation. It was found that the delay at the collar signals was not sufficiently high (about 3 min) to induce mode shift to buses. This was due to the limited storage space of about 800 m (or 100 veh) and hence limited queue lengths and delay at these signals. The bus journey times between residential zones and the city centre was reduced by less than 1 min. The experiment was not regarded as successful in achieving travel demand reduction, but the principle of gating traffic by means of signals was well demonstrated.

In strategic queue management a 'minor' road where there is limited queue space may receive priority treatment in phase and offsets. For example, in the case of two intersecting arterial roads, one should doubt the wisdom of always giving priority to the city-bound route: The consideration of total network performance may suggest that more green time be allocated to the cross route if necessary to minimise blockage effects there.

In summary, one needs to query the traditional approach of allowing a UTC system to distribute green times according to prevailing conditions. It may be necessary to design signal settings according to queue storage space or other strategies, especially in oversaturated conditions. Congestion should not be seen as something that is to be eliminated. It could be designed into an urban traffic system to establish equilibrium.

3. ON-LINE QUEUE MONITORING

A first step towards queue management is to monitor where the queues are. The monitoring process should be an integral part of a UTC system so that queue lengths at all the intersections under its control are known every cycle *on-line*. The queue lengths are to be calculated from traffic flow data and signal settings of the *previous* cycle with a queue formation and dissipation model, and there is no need for the prediction of parameters in the calculations.

3.1 Modelling Queue Formation and Dissipation

The traditional model for queue formation and dissipation is recommended as the basis for on-line queue monitoring. This model is illustrated in Fig. 2. A detector at a distance L from the stopline provides the arrival flow profile. If L is at a location beyond the maximum queue length at D as shown, a complete arrival profile is available for queue estimation. If the sensor is at the stopline ($L = 0$), an accurate departure profile is available and there is no need to assume a constant departure or saturation flow rate.

The triangle ABD in Fig. 2 represents the total delay (in veh-s). The queue at the start of a green phase is represented by BR. The maximum back of queue is at D (or 6 veh in Fig. 2); this is the queue that forms at the back as vehicles at the front begin to move off when signal turns green.

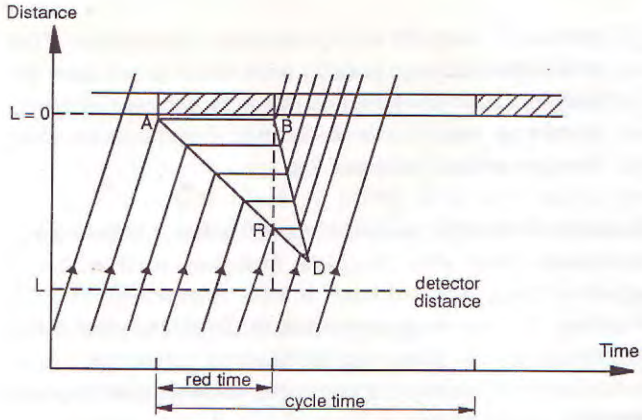

FIGURE 2: Delay and Queue Lengths at a Stopline

The accuracy level of queue estimation obviously depends on the number of detectors and their locations. High accuracy can be achieved with two detectors per lane – one at the stopline and the other beyond the maximum back of queue. However, most UTC systems use only one detector per lane (and some use one detector for several lanes), largely due to the cost of installing and maintaining inductive loop sensors. An important issue is therefore the accuracy level that can be achieved by one set of detectors per approach. Empirical studies undertaken to determine the accuracy of queue estimation in SCATS and TRACS are described below.

3.2 Queue Estimation in SCATS

The operation of SCATS relies on detectors located near stoplines to measure parameters such as traffic flow, occupancy and space times, and the degree of saturation. It does not rely on the modelling of queue lengths. Luk, et al. (1988) proposed a scheme whereby estimated queue lengths could constitute a useful part of the current SCATS monitoring facilities. This scheme makes use of the following data:

- green and red phase times at the end of each cycle,
- departure flow profiles from stoplines detectors, and
- estimated arrival flow profiles based on the turning flows at the upstream intersection.

This proposed scheme makes use of an accurate stopline departure profile, and should be particularly useful in congested conditions when the saturation flow varies rapidly and cannot be assumed constant. The scheme was tested on a road link in Box Hill, Melbourne. The two intersections were under SCATS control and vehicles arrived in platoons. Data loggers were connected directly to vehicle detectors at the two signal controllers to retrieve the stopline traffic flow profiles (SCATS currently does not transmit traffic flow data from a controller to the central computer in time steps of one or several seconds).

The results of comparing observed and predicted queue lengths are summarised in Table 1. Both the queue at green start (QGST) and the maximum back of queue (QMAX) were manually measured. There was more uncertainty in estimating the maximum back of queue due to the slowing down of a vehicle as it joins a queue. For QMAX, the R^2–terms were better than 0.50; for QGST, the R^2–terms were 0.70 and 0.79. Although the percentage differences of the means for QMAX were better than 9 per cent, the root mean squared differences (RMSD) were of relatively higher values than those of QGST.

While this study establishes the practicality of on-line queue in an UTC system where detectors are near the stoplines, it appears that the traditional queue modelling method may not lead to a high level of estimation accuracy.

3.3 Queue Estimation in TRACS

The Queensland Department of Transport recently took up the challenge of developing the performance monitoring software as additional features of its control system, TRACS. Apart from queue lengths, other performance indices include throughput, delay, stop and the degree of saturation.

TABLE 1: Comparing Observed and Estimated Queue Lengths in SCATS

	Mean observed queue (veh)	Mean SCATS queue (veh)	Percentage difference (%)	Root mean squared diff. (veh)	R^2 –terms
Peak Period					
Queue at green start	19	16	–16	8	0.70
Maximum back of queue	22	23	2	12	0.64
Off-peak period					
Queue at green start	11	9	–23	4	0.79
Maximum back of queue	13	15	9	7	0.53

In TRACS, the detectors are usually located at 35 m from the stopline on main road approaches, and near the stopline for minor approaches or sidestreets. The mode of operation is timing plan selection according to prevailing occupancy of flow levels measured by the loop detectors.

The queue estimation software is based on the same model of queue formation and dissipation in Fig. 2. It has to fit into the current TRACS structure and some of the constraints are as follows:

(a) Undercounting of traffic flow may occur because each loop sensor is scanned once every 2 s to determine whether it is occupied or not;

(b) Vehicles in a queue are assumed to depart from a stopline at a rate of 1 veh per 2 s; and

(c) When queue lengths exceed beyond 35 m (or 6 veh), the maximum back of queue is identified by analysing the headways after the green phase starts. The reliability of this method is affected by the low sampling rate described in (a).

Given these constraints, a study was carried out in Chapel Hill, Brisbane , to ascertain the level of accuracy that can be achieved with this version of software (Luk 1991). Typical results for platooned arrivals (arterial approaches) and random arrivals (sidestreet approaches) are shown in Table 2.

The results are consistent with those obtained in SCATS. The major findings are:

(a) For platooned arrivals, the R^2–terms were 0.73 for QGST and 0.64 for QMAX, and were similar to the SCATS results;

(b) It was more difficult to estimate QMAX than QGST for platooned arrivals, as indicated by the relatively larger values of RMSD;

(c) For random arrivals (from a freeway exit at the study site), the accuracy level was lower as indicated by smaller R^2–terms and percentage differences. This was because queue lengths were often longer than the loop distance of 35 m, and queue length estimation by headway analysis was limited by the slow data transfer rate of once per 2 s.

Results from the monitoring of other performance indicators were also found to correlate well with the observed data. The software is now released in TRACS Version 2.4 for on-going tests in Brisbane and other provincial cities. Plans are in hand to refine the queue model and to modify the controller telemetry to improve the data transfer rate.

4. DISCUSSION

The two empirical studies have confirmed the practicality of on-line queue monitoring as part of a UTC system. The level of accuracy is not expected to be high – the difference between the observed and estimated means is about 10 to 30 per cent and the R^2–term is about 0.50. This should not be surprising if traffic counts from loop sensors are currently measured with an error of about ± 5 per cent, and there is no simple way to consider the effect of heavy vehicles on-line. In oversaturated conditions, the traditional method of queue modelling with data from one detector per lane is unlikely to achieve much higher accuracy than those described in this paper. Information from an adjacent detector of the upstream intersection should also be used in identifying blocking effects; or else the obvious solution of installing two detectors per lane should be considered.

Some theoretical work at ARRB on paired intersection modelling investigated the blocking effect of the downstream queue (Rouphail and Akcelik 1991). The results so far confirmed some of the practices such as simultaneous offsets and gating. More research is required to introduce the theory into an on-line monitoring framework.

Other methods for queue monitoring have been reported in the literature. These include the following:

TABLE 2: Comparing Observed and Estimated Queue Lengths in TRACS

	Mean observed queue (veh)	Mean TRACS queue (veh)	Percentage difference (%)	Root mean squared diff. (veh)	R^2 –terms
Platooned arrivals					
Queue at green start	3.0	3.1	5	1.6	0.73
Maximum back of queue	3.6	4.5	27	3.5	0.64
Random arrivals					
Queue at green start	6.7	4.9	– 27	3.4	0.40
Maximum back of queue	7.3	5.0	– 31	4.0	0.42

(a) Regression – it is possible to estimate queue lengths, say, every 15 min by relating queues to mean values of signal settings and traffic parameters such as flow and occupancy. This was investigated as part of the TRACS study, but this approach cannot model the cycle-by-cycle blocking effect. It is still useful for presenting a macroscopic picture of where the queues are *on average* (Luk 1991).

(b) Modelling by discretisation in time and space – Michaelopoulos (1988) reported the principle of modelling vehicle movements in small time steps of several seconds and in small space segments of about 10–50 m. The principle makes use of the fundamental relation: flow = density x speed, and requires the conservation of flow. The method has the benefit of explicitly modelling the spatial domain and hence the downstream disturbance on traffic flow. The limitation appears to be the large amount of information required on the boundary. If the boundary conditions are not met, the conservation of flow will not hold and the accuracy of modelling is uncertain.

(c) Image processing – the use of image processing techniques for traffic engineering applications has often been reported. Each pixel of a video frame emulates a sensor and a large number of pixel-sensors are therefore available for vehicle detection. Windows are created in a framework to reduce the amount of real-time processing. Rourke and Bell (1991) reported the feasibility of identifying the presence or absence of a queue by determining the amount of movement in a lane represented by a window of pixels. Apart from the high cost of equipment for areawide applications, other technical difficulties include: erroneous detection due to the shadow of adjacent lane traffic, appropriate locations for the camera, cloud and weather pattern effect, and night time operation.

'Tagging' vehicles with transponders has also been proposed as a means of monitoring congestion (Longfoot and Quail 1990). The signal controller at each intersection receives vehicle identification codes from the onboard transponders. Travel times between two intersections are determined from the times that a vehicle passes through each intersection. Note that the travel time is not a good indicator of queue interactions, i.e. a long travel time (and hence delay) may not be due to blocking effects. The accuracy of this indicator is also dependent on the number of vehicles equipped with transponders, and on how many of these tagged vehicles travel along a particular route at a particular time.

The on-line modelling of queue formation and dissipation is therefore a fundamental step towards queue management by means of a UTC system. It is a natural extension of the function of the system. The concept can be implemented in a fixed-time or adaptive system. It does not involve the issue of privacy intrusion as a result of tagging vehicles. Queue information is available 24 hourly independent of the number of vehicles in the road system. On-going research in Australia and overseas is in-hand to refine the model and to develop better and more reliable detectors.

5. CONCLUSIONS

This paper highlights the significance of queue management in the operation of a signal control system in a congested city. In a network where the queue storage space is scarce, a control philosophy of reducing delay may no longer be appropriate. Congestion, and traffic queues, are now essential parts of a traffic system. Queues should be managed so that a traffic system becomes least efficient. On-line queue monitoring is a first step towards the implementation of strategic queue management. The accuracy of queue estimation in SCATS and TRACS based on the traditional queue formation and dissipation model is about 10–30 per cent in estimation errors, with R^2–terms of about 0.50. This level of accuracy can be improved with more refined modelling, more detectors, and higher data transfer rate from controllers to the central computer. It is recommended that on-line monitoring facilities be implemented as a natural extension of the functions of a UTC system.

6. ACKNOWLEDGEMENTS

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STUDY ON FACTORS CAUSING RUTTING OF PAVEMENT AND DESIGN OF SURFACE COURSE

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ABSTRACT

Reports are made in this paper on the results of the analyses carried out on the data obtained in tests which were implemented with the aim of evaluating the resistance to plastic flow of surface course asphalt mixtures generally found on expressways in Japan today using their dynamic stability as an index, together with the state of the investigations concerning resistance to plastic flow being carried out at present by the Japan Highway Public Corporation.

The main conclusions reached in these studies were as follows.

1. Plastic flow resistance cannot be expected in asphalt mixture used in the abrasion area.
2. The following considerations should be made in order to raise plastic flow resistance.
   a. Asphalt with low penetration should be used.
   b. The granular materials in the aggregate should be reduced.
   c. The number of blows in the Marshall test should be 75 times.
3. Dynamic stability of over 2,000 cannot be expected in mixtures using straight asphalt. Modified materials should be used on routes where higher stability is required.

1. PRESENT STATE OF EXPRESSWAYS IN JAPAN

The total length of expressways in Japan will reach 5,000km by the end of 1991 and construction continues at a pace of over 200km a year. An average of 2.76 million vehicles a day used these expressways in 1990 and they have become indispensable not only for industrial activities of transportation of goods and commercial traffic but also for both daily life and leisure activities, as the main arteries for transportation in Japan.

The increase in the traffic volume and the aging of the road, on the other hand, have led to an increase in the cost of maintaining the pavements, which in 1990 reached ¥ 20 billion (Figure 1) or nearly 20% of the total expenditure for improvement of expressways.

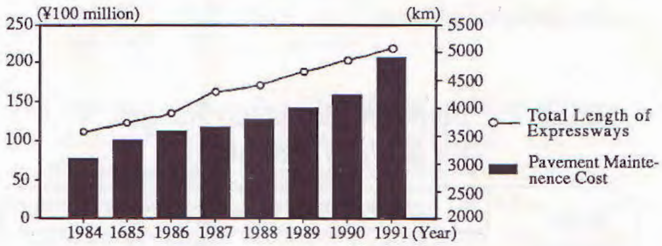

FIGURE 1: Total Length of Expressways in Service and Maintenance Costs

Asphalt pavements account for 90% of all the pavements on expressways in Japan and, of the causes that give rise to the need for maintenance work such as rutting and cracking, rutting is the most frequent.

It is needless to say that the pavements which are actually in contact with the tyres of the cars play an important role in ensuring safe and comfortable high-speed travel. At the same time, traffic congestion accompanying maintenance work has become a serious social problem, and there is today an urgent need for creation of asphalt pavements that are resistant to rutting.

Rutting can be divided into plastic flow rutting due to the load of the traffic and abrasion rutting due to wear, for example, by tyre chains. Whether the expressway in question is in an area where roads are subject to abrasion and whether the expressway will be subject to traffic of heavy vehicles are important factors in designing expressways. Under the Design Manual for Asphalt Pavements in force at present, the whole of Japan is divided into zones as shown in Figure 2 and the mix standards laid down for these zones are as shown in Tables 1, 2 and 3.

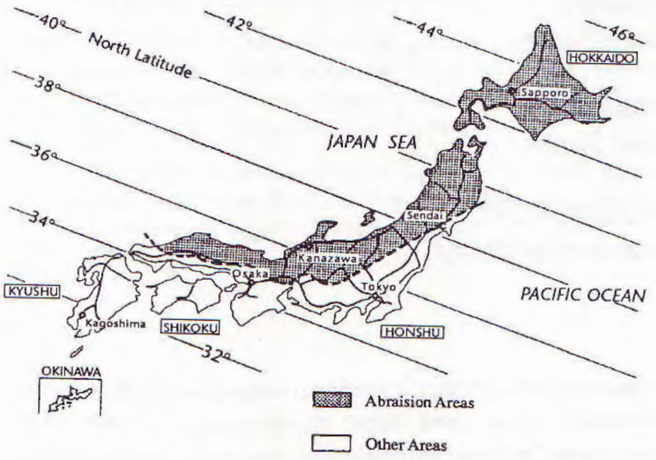

FIGURE 2: Two Zones for Mix Proportion Design

Studies on the damage to asphalt pavements in recent years, however, indicate that plastic flow rutting is occurring also in abrasion areas and areas of medium-light traffic and plastic flow rutting becomes especially prominent after repair works on the surface course, giving rise to a need to review the mixing methods for the surface course asphalt mixtures.

TABLE 1: Standard Gradation Ranges for Surface Course

Sieve Opening (mm)	Percent Pass by Weight		
	Type I 13 mm	Type II	
		13 mm	20 mm
25			100
20	100	100	95 to 100
13	95 to 100	95 to 100	75 to 95
10	75 to 95	75 to 95	68 to 88
5	55 to 75	52 to 72	
2.5	38 to 58	40 to 60	
0.6	21 to 36	25 to 45	
0.3	13 to 25	16 to 33	
0.15	6 to 16	8 to 21	
0.075	4 to 8	5 to 12	

Reported below are the results of wheel-tracking tests on surface course mixtures using straight asphalt and the present state of the investigations being carried out at the Japan Highway Public Corporation Laboratory on the mix design with the aim of raising their resistance to plastic flow.

TABLE 2: Standard Proportion for Surface of Asphalt Mixtures

Types of Surface Course Mixture	Type I		Type II	
Area Zone	Ordinary Area		Abrasion Area	
Traffic Category	Heavy	Medium -Light	Heavy	Medium -Light
Grade of Asphalt Penetration (1/10mm)	60 to 80	80 to 100		
No.of Blows in the Marshall Test (each face)	75	50		

NOTE: The traffic category is that volume of heavy vehicles during the first year.

Heavy traffic is over 3,000 cars/day/one direction.

Medium-light traffic is less than 3,000 cars/day/one direction.

TABLE 3: Standard Values For Marshall Test

ITEM	SURFACE COURSE
Stability (kg)	600 or more
Flow Value (1/100cm)	20 to 40
Void Ratio (%)	3 to 5
Voids Filled (%)	75 to 85
Residual Stability after Immersion (%) 60°C, 48 hours	75 or more
Field Compaction in Relation to Marshall Standard Density (%)	96 or more

2. OUTLINE OF WHEEL-TRACKING TEST

2.1 Test Method

The wheel-tracking test was developed at the British RRL (present day TRL) as a method of studying rutting on asphalt pavements. The rutting and kneading action on actual roads under traffic of heavy vehicles in high temperatures are simulated for laboratory evaluation of the resistance of asphalt mixtures to plastic flow. A modified version of the British test method, adjusted to suit the climatic conditions in Japan, has been used by the Japan Highway Public Corporation since 1986.

The tests are carried out on special testers (Figure 3). A solid tyre under a constant load and with a predetermined contact pressure and rubber hardness is made to travel to and fro over the test piece and the displacement in the part over which the tyre travels during a given time is measured to evaluate the dynamic stability DS (cycles/mm). The temperature at which the tests are carried out, which in Britain is 45°C, is increased to 60°C allowing for the maximum road surface temperature in Japan.

The test conditions used by the Japan Highway Public Corporation are given in Table 4.

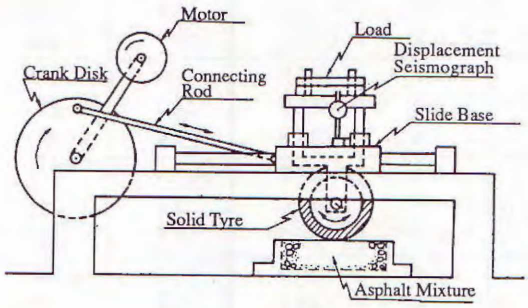

FIGURE 3: Wheel-Tracking Tester (Crank type)

TABLE 4: Conditions for Wheel-Tracking Test

ITEM	CONDITION
Type of Testing Machine	Crank
Type of Tire	Solid Tire
Hardness of Rubber	JIS Hardness 78 ± 2
Traveling Speed (cycles/min)	42 ± 1
Wheel Load (kgf)	70 ± 1
Testing Temperature (°C)	60
Specimen Size (mm)	300 x 300 x 50

2.2 Determination of Mix Conditions for Asphalt Mixture

As mentioned above, the surface course mixtures with straight asphalt that are used at present on expressways are of two types, Type I used in ordinary areas and Type II used in abrasion areas. The values for DS used in the past were around 500 for medium-light traffic sections and 1,000 for heavy traffic sections of Type I, and around 200 for Type II.

A total of 53 types of mixtures with different combinations of various items as described below, in some cases going beyond the normal ranges, were used in the tests.

- **Number of blows in Marshall Test:** 50, 75 and 100
  The standard is 50 or 75 times. Mixtures with high densities were tested with the addition of 100 times.
- **Maximum particle size of coarse aggregate:** 13mm for Type I, 13 and 20 mm for Type II
- **Asphalt penetration:** 40 to 100 (1/10mm)
  The normal ranges are 60 to 80 and 80 to 100. The range 40 to 60 was added to test mixtures with particularly hard asphalt.
- **Weight percentage passing 2.36 mm sieve:** 36 to 56%

It is the standard practice to express the particle size distribution characteristics of the aggregate in the mixture in terms of the weight percentage of the aggregate passing through a 2.36 mm sieve.

The optimum asphalt content obtained in the Marshall test was used for the asphalt content.

2.3 Test Results

The test results for the different mixtures are shown in Table 5.

The Regression Equation (1) was obtained with multiple correlation coefficient of 0.929 in the multiple regression analysis implemented with the DS as the object variable (Y) and the number of blows (X_1), maximum particle size (X_2), penetration (X_3) and weight percentage passing 75 μ sieve (X_4) as explanatory variables.

When there was a strong correlation between the items as shown in the table of correlation coefficients (Table 6), the problem of multicolinearity was avoided by selecting only one of the explanatory variables with consideration for their correlation with DS.

$$\begin{aligned} Y = 11.16X_1 &+ 0.1376X_2 - 29.54X_3 - 172.0X_4 + 3421 \quad (1) \\ (0.284) \quad &(0.001) \quad (-0.552) \quad (-0.428) \end{aligned}$$

The figures shown in brackets in the equation above are the standard partial regression coefficients (regression coefficients obtained using the data transformed so that the average = 0 and the variance = 1) and the effect of each explanatory variable can be evaluated from their absolute values. In Equation (1), the value for the maximum particle size is very close to 0. This is thought to be because 20 mm aggregate is found only in Type II mixtures. The Regression Equation (2) was obtained with the multiple correlation coefficient at 0.929 by excluding the value for the maximum particle size.

$$\begin{aligned} Y = 11.16X_1 &- 29.54X_3 - 171.8X_4 + 3422 \quad (2) \\ (0.284) \quad &(-0.552) \quad (-0.427) \end{aligned}$$

This confirms the obvious fact that the DS will be large when the number of blows in the Marshall test is large, the asphalt penetration is small and the proportion of the granular materials is low.

Similar multiple regression analyses were carried out for each type.

TABLE 5: Wheel-Tracking Test Results

Mix Number	Mixture Type	Asphalt Type	Number of Blows	Grading	Asphalt Penetration 1/10 mm	Asphalt Content %	2.36mm Pass %	0.075mm Pass %	Dynamic Stability Cycles/mm
1				40	85	5.6	39.9	7.3	215
2				44	85	5.8	43.9	7.8	160
3			50	*48	85	6.0	48.0	8.4	145
4				52	85	6.2	52.1	8.9	125
5				56	85	6.4	56.3	9.5	110
6		80/100		40	85	5.4	39.9	7.3	295
7				44	85	5.6	43.9	7.8	235
8	Type II (13)		75	48	85	5.8	48.0	8.4	155
9				52	85	6.0	52.1	8.9	130
10				56	85	6.2	56.3	9.5	120
11				40	77	5.6	39.9	7.3	395
12			50	48	77	6.0	48.0	8.4	220
13				56	77	6.4	56.3	9.5	160
14		60/80		40	77	5.4	39.9	7.3	645
15			75	48	77	5.8	48.0	8.4	275
16				56	77	6.2	56.3	9.5	225
17				40	8.5	5.4	39.9	7.3	215
18				44	85	5.6	43.9	7.8	175
19			50	*48	85	5.8	48.0	8.4	155
20				52	85	6.0	52.1	8.9	130
21		80/100		56	85	6.2	56.3	9.5	110
22				40	85	5.0	39.9	7.3	335
23				44	85	5.2	43.9	7.8	255
24	Type II (20)		75	48	85	5.4	48.0	8.4	215
25				52	85	5.6	52.1	8.9	185
26				56	8.5	5.8	56.3	9.5	165
27				40	77	5.4	39.9	7.3	355
28			50	48	77	5.8	48.0	8.4	290
29		60/80		56	77	6.2	56.3	9.5	175
30				40	77	5.0	39.9	7.3	595
31			75	48	77	5.4	48.0	8.4	440
32				56	77	5.8	56.3	9.5	315
33				36	87	5.9	35.7	4.9	630
34				40	87	6.0	39.9	5.4	530
35			50	*44	87	6.1	44.0	6.1	450
36				48	87	6.2	48.2	6.6	380
37		80/100		52	87	6.3	52.4	7.3	360
38				40	87	5.7	39.9	5.4	840
39			75	44	87	5.8	44.0	6.1	710
40				48	87	5.9	48.2	8.6	520
41			100	44	87	5.6	44.0	6.1	1090
42				40	64	6.0	39.9	5.4	930
43	Type I (13)		50	44	64	6.1	44.0	6.1	690
44				48	64	6.2	48.2	6.6	510
45				36	64	5.6	35.7	4.9	1510
46		60/80		40	64	5.7	39.9	5.4	1220
47			75	*44	64	5.8	44.0	6.1	1090
48				48	64	5.9	48.2	6.6	1010
49				52	64	6.0	52.4	7.3	900
50			100	44	64	5.6	44.0	6.1	2170
51				40	51	5.7	39.9	5.4	2570
52		40/60	75	44	51	5.8	44.0	6.1	2010
53				48	51	5.9	48.2	6.6	1630

* *Mixtures generally used under present mix standards.*

TABLE 6: Correlation Coefficients

	Number of Blows	Max. Aggregate Size	Asphalt Penetration	19.0 mm Pass	13.2 mm Pass	9.5 mm Pass	4.75 mm Pass	2.36 mm Pass	0.6 mm Pass	0.3 mm Pass	0.15 mm Pass	0.075 mm Pass	Asphalt Content	DS
Number of Blows	1.000													
Maximum Aggregate Size	- 0.099	1.000												
Asphalt Penetration	- 0.230	0.252	1.000											
19.0 mm Pass	0.000	0.000	0.000	1.000										
13.2 mm Pass	0.098	- 0.987	- 0.249	0.000	1.000									
9.5 mm Pass	0.124	- 0.909	- 0.317	0.000	0.944	1.000								
4.75 mm Pass	0.037	- 0.107	- 0.095	0.000	0.200	0.464	1.000							
2.36 mm Pass	- 0.062	0.176	0.156	0.000	- 0.084	0.095	0.858	1.000						
0.6 mm Pass	- 0.087	0.251	0.223	0.000	- 0.164	- 0.021	0.769	0.988	1.000					
0.3 mm Pass	- 0.126	0.361	0.320	0.000	- 0.285	- 0.207	0.583	0.918	0.968	1.000				
0.15 mm Pass	- 0.132	0.387	0.344	0.000	- 0.318	- 0.258	0.526	0.888	0.948	0.996	1.000			
0.075 mm Pass	- 0.151	0.442	0.393	0.000	- 0.384	- 0.370	0.378	0.800	0.883	0.972	0.985	1.000		
Asphalt Content	- 0.438	- 0.422	- 0.025	0.000	0.498	0.641	0.759	0.615	0.538	0.389	0.333	0.219	1.000	
DS	0.476	- 0.356	- 0.785	0.000	0.341	0.402	- 0.040	- 0.408	- 0.495	- 0.611	- 0.640	- 0.687	- 0.140	1.000

High Positive Correlation

High Negative Correlation

Low Correlation

a. **Type I** (multiple correlation coefficient: 0.903)

$$Y = 4.058X_1 - 26.75X_2 - 1336X_5 + 10560 \quad (3)$$
$$(0.095) \quad (-0.599) \quad (-0.635)$$

b. **Type II** (multiple correlation coefficient: 0.906)

$$Y = 0.9786X_1 - 19.92X_2 - 220.5X_5 + 3081 \quad (4)$$
$$(0.105) \quad (-0.604) \quad (-0.449)$$

where X_5: asphalt content

The same items (the number of blows in the Marshall test, asphalt penetration and asphalt content) were used as explanatory variables for both *a* and *b*. While there was a strong correlation (correlation coefficient: approx. 1.0) between the particle sizes, between the asphalt content and particle sizes the correlation was low for Type I and high (correlation coefficient: 0.8) for Type II. The results, in other words, indicate that the DS can be raised by setting the asphalt content slightly below the optimum value obtained in the Marshall test in the case of Type I and by reducing the granular materials in the aggregate and so reducing the asphalt content in the case of Type II.

In any case, although it is possible to increase the dynamic stability by adjusting the asphalt penetration and content, it is clear that one cannot expect resistance to plastic flow in the Type II mixtures used at present for abrasion areas and Type I mixtures for medium-light traffic. Where marked plastic flow rutting is observed in such areas in future, mixtures with a larger number of blows, smaller penetration and lower proportion of granular materials than in conventional mixtures should be selected.

On routes subject to heavy traffic, the plastic flow resistance can be raised by using asphalt with penetration values of below 60.

3. INVESTIGATIONS ON RESISTANCE TO PLASTIC FLOW

The present state of the investigations on plastic flow resistance at the Japan Highway Public corporation is discussed below.

3.1 Analysis of Causes of Rutting

Analyses have been carried out using the records of construction works on expressway pavements and property data on surface conditions of roads in service stored in the data bank of the Japan Highway Public Corporation's pavement management system, with the progress of rutting as the object variable and with factors considered to be the causes of rutting, such as the mix conditions, climatic conditions and traffic volumes, as the explanatory variables.

Analyses are also being carried out for determination of zones from the conditions of damage, traffic volumes and climatic conditions on expressways in service, in order to decide whether more importance should be attached to the resistance to plastic flow or resistance to wear in accordance with the location and uses of the routes.

3.2 Test Construction According to Provisional Standards

Provisional standards for pavements have been proposed for types of roads classified according to volume of heavy traffic and conditions of damage (wear and plastic flow), and adding DS as an index, and test construction is being carried out in such a way as to allow comparison with conventional standards. Preparations are being made at the Japan Highway Public Corporation Laboratory for analysis of the data collected.

It is hoped that the analysis work will be of help in checking the properties actually obtained with the mixtures and pavements constructed with the DS values as determined for various areas and routes as their target values, as well as in assessing their effects with regard to the plastic flow resistance.

3.3 Tests on Effects of Modified Asphalt

It has been confirmed that one cannot hope for DS values in excess of 2,000 with standard mixtures using straight asphalt. On routes subject to heavy traffic, the rapid progress of rutting may mean that DS of this level may not provide much resistance against plastic flow. It has become necessary in recent years as a result to use modified asphalt in order to obtain mixtures with DS of over 3,000 on certain routes. Because the large variety of modified materials used means that there is much uncertainty about their properties and the properties of mixtures containing them, physical tests are being carried out on the products on the market at present, along with laboratory tests to assess the basic properties of the mixtures.

3.4 Research on Composite Pavement

In composite pavements, cement concrete materials used in the binder and base courses of roads and asphalt concrete in the surface course, making use of the advantages of the different types of concrete. A 9 km section of the Sanyo Expressway has been constructed with composite pavements and the plan is to conduct studies on their properties over a period of 10 years.

3.5 Introduction of Semi-Flexible Pavement

In semi-flexible pavements, cement milk is injected into the voids in open graded asphalt with its large voids. This results in extremely high resistance to plastic flow and these pavements are beginning to be used on climbing lanes and in parking areas.

4. CONCLUSION

The items that need to be considered in the mix design of surface course asphalt mixtures have changed with time. There were times in the past when the most important considerations were for measures to prevent cracking or flush. While the mix procedures have been revised in accordance with the changing needs, excesses in certain directions have tended to result in other kinds of damage.

Rutting due to plastic flow has become the most serious problem today, requiring an urgent response, and this is the reason for publication of test results at this stage. Such investigations had been made in the past, when an attempt to solve the problem by increasing the proportion of coarse aggregates and using low-penetration asphalt resulted in increased generation of cracks. Another problem at present is that the low accuracy of the results of wheel-tracking tests does not allow the adoption of dynamic stability as the standard values. The authors hope to establish the mixes for surface course asphalt mixtures with high durability through investigations for raising the accuracy of wheel-tracking tests, and revision of the provisional standards on the basis of laboratory tests on surface course asphalt mixtures and test construction on expressways in service.

5. REFERENCES

(1) Express Highway Research Foundation, **"Report of Investigations on Study and Test Methods Relating to Pavements"** 1985. pp.17

(2) Japan Highway Public Corporation, **"Design Manual for Asphalt Pavements"** 1983, pp. 4-26

SIDRA AS AN ADVANCED TRAFFIC SIGNAL ANALYSIS METHOD FOR THE HIGHWAY CAPACITY MANUAL

by

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ABSTRACT

This paper discusses the Highway Capacity Manual (HCM) version of the **SIDRA** program developed by the Australian Road Research Board as an aid for capacity, timing and performance analysis of signalised and unsignalised intersections. Various aspects of the signalised intersection analysis methodology described in the HCM and implemented in the Highway Capacity Software (HCS) package, as well as proposals for improvement published in various articles, are discussed. While various features of the HCM have been incorporated into SIDRA and made available to all SIDRA users, many features of SIDRA are offered under the HCM option as enhancements to the HCM/HCS methodology. An application of the SIDRA HCM option to an example from the HCM is given. The HCM version of SIDRA is fully developed for use by practitioners who may wish to take advantage of the more detailed analysis method offered by SIDRA (analysis in US or metric units is allowed). SIDRA can be used in the standard Australian way, or can be calibrated for local conditions in different countries through its Default Values File without any program change.

1. INTRODUCTION

SIDRA (Signalised and unsignalised Intersection Design and Research Aid) is a software package developed by the Australian Road Research Board as an aid for *capacity, timing and performance* analysis of signalised and unsignalised intersections (including roundabouts). First released in 1984, SIDRA was in use for *practice, research and teaching* in about 210 organisations/sites in 36 countries as of July 1992.

With input and output facilities at individual turn, lane, lane group, approach road, movement grouping and intersection levels, SIDRA provides a flexible structure which allows multilevel analysis of very simple to very complex intersection conditions. The ability of the user to *calibrate* SIDRA models for local conditions is an important feature of SIDRA.

The capacity and timing analysis methods in SIDRA have evolved from the ARRB Research Report ARR No. 123 (1), and are now substantially more advanced. Extensive documentation is available on SIDRA (2-14). General reviews of Australian research on traffic signals are also available (15,16).

The 1985 Highway Capacity Manual (HCM) (17) introduced various improvements to the existing signalised intersection analysis methods, and brought the Australian and U.S. methodologies closer together. Many suggestions have been made to further improve the methodology for signalised intersection analysis described in the HCM and implemented in the Highway Capacity Software (HCS) package (18), and work is in progress towards this end (19-37).

This paper presents information about the HCM version of SIDRA and discusses various aspects of the HCM/HCS methodology as well as the proposals to modify it. The discussion is limited to signalised intersections. An application of SIDRA to an example from the HCM is presented. For more detailed information on the HCM version of SIDRA, the reader is referred to SIDRA document DN 1709 (14).

2. THE HCM VERSION OF SIDRA

A Highway Capacity Manual version of SIDRA has been developed (11-14) in order to facilitate the comparison of the SIDRA and HCM methodologies, and to incorporate some useful features of the HCM into SIDRA for the benefit of its users. In turn, this has resulted in identifying various areas for potential improvement in the HCM methodology which should benefit the users and developers of the HCM and HCS.

Although the HCM version of SIDRA was basically developed for research purposes, it is fully developed for use by practitioners who may wish to take advantage of the more detailed analysis method offered by SIDRA (analysis in US or metric units is allowed).

The HCM version of SIDRA answers many suggestions put forward by US researchers and practitioners for the improvement of the HCM methodology (19-37). It uses the capacity (basic saturation flow adjustment factors) and delay model parameters of the HCM in order to calibrate the general SIDRA models. This is done through a Default

Values File (11). Detailed user guides and technical notes are available for the users of the HCM version of SIDRA (11,14).

Progression factors by arrival type for the effect of platooned arrivals on delay, the use of peak hour factor, and models for conflicting pedestrian volumes, parking and bus effects on saturation flows are the main features of the HCM that have been incorporated into SIDRA and made available to all SIDRA users. An output table which presents capacity, degree of saturation (*v/c ratio*), delay and level-of-service information in the HCM/HCS style is available. Graphics output program GOSID displays delays and levels of service in a picture form in the HCM version of SIDRA.

Various features of SIDRA which are considered to be enhancements to the HCM/HCS methodology are described in the following sections.

2.1 Lane-by-Lane Analysis

Lane-by-lane level of detail used for capacity and performance (delay, queue length, etc.) modelling is a fundamental feature of SIDRA. The lane-by-lane method, which is also used in the Canadian and Swedish methods (38,39), offers many advantages over the *lane group* method which is used in the HCM/HCS as well as the U.K. OSCADY method (40). Eliminating the need to combine various approach lanes as lane groups provide freedom in movement description, and ensures more accurate and, in principle, simpler modelling. However, SIDRA identifies lane groups for the purpose of modelling lane utilisation.

2.2 Lane Flows and De Facto Lanes

SIDRA carries out lane flow calculations as an integral part of the capacity estimation process. Effective (*de facto*) exclusive lanes are established explicitly as part of this process. Lane under utilisation factors, parking and bus parameters, etc. can be specified by the user for individual lanes. Explicit modelling of lane flows helps the traffic engineer to design efficient lane arrangements.

Various problems of the HCM/HCS methodology in determining de facto exclusive lanes have been discussed in an unpublished note by the author (41). A limitation of the HCM method is to assume a right turn adjustment factor of 1.0 in lane flow calculations, neglecting the effect of conflicting pedestrian flows.

Some incorrect solutions have been obtained from the HCS package in heavy right turn volume cases. For example, consider the case where cycle time = 90 s, green time = 45 s, three lanes with basic saturation flow of 1800 tcu/h, through (T) volume of 400 veh/h, right-turn (R) volume of 600 veh/h, and ideal conditions except for a conflicting pedestrian volume of 400 ped/h.

The results summarised in Table 1 show the contrast between the HCS and SIDRA results. This example demonstrates an important problem with capacity analysis methods based on lane groups which give no information to the user about the individual lane flows and effective lane disciplines.

In Case (a), the HCS solution implies that right-turns can depart from 3 lanes (which may not be possible

TABLE 1: Comparison of SIDRA and HCS Results for a Simple Example for *De Facto* Lane Identification with Heavy Right Turn Volumes

	SIDRA Solution			HCS Solution		
Specified Lane Dis.	Effective Lane Dis.	Deg.of Sat.,x	Delay d (sec)	Implied Lane Dis.	Deg.of Sat.,x	Delay d (sec)
Case (a)						
T	T	0.222	9.8	T R }		
T	T	0.222	9.8	R }	0.465	11.3
T R	R	1.011	48.5	R }		
			33.0			
Case (b)						
T	T	0.444	11.4	T }	0.222	9.6
T R	R	0.505	12.2	T }		
R	R	0.505	12.2	R	1.011	48.4
			11.9			32.9

physically), and this is in spite of the user specification of right turns from Lane 3 only. In case (b), the HCS solution identifies Lane 2 as a de facto exclusive *through* lane, whereas the SIDRA solution indicates that Lane 2 should be a de facto exclusive *right-turn* lane.

In both cases, the two solutions are drastically different in terms of delays and degrees of saturation. The HCS solutions are incorrect in both cases. In Case (a), the SIDRA solution indicates that no through vehicle would benefit from going into Lane 3 (higher x, higher delay). The HCS solution indicates a satisfactory design, but it does not relate to the lane arrangement specified by the user. In fact, the specified lane arrangement would cause serious problems in practice if adopted for design. In case (b), no through vehicle would benefit from going into Lane 2, but the lane flows are fairly well balanced, yielding a low degree of saturation. On the other hand, the HCS solution suggests that the specified lane arrangement will give an unacceptable solution. Contrary to the HCS solution, right-turns should be allowed to go into Lane 2 as specified by the user.

2.3 Two Green Periods

The use of two green periods with different saturation flows is essential for accurate modelling of protected-permitted turns (1,21) as well as slip lanes and turns on red. Much development effort has gone into modelling capacity and performance, determining critical movements (periods) and computing signal timings with two green periods per cycle, and the result have been incorporated into SIDRA.

The alternative method of combining the two green periods with an average saturation flow as in the U.K. OSCADY program (40) is not considered to be satisfactory for modelling capacity and performance. The HCM method for protected-permitted turns has various limitations (5,6,9,21,23,33), and the HCS method of providing the user with options to split arrival volumes between the two green periods does not seem to give a satisfactory result although it is an improvement over the original HCM methods.

2.4 Permitted (Opposed) Turns

A unique feature of SIDRA is to model opposed (permitted) turns by *lost time method* which treats blocked intervals as effectively red (1,9,11). This method usually results in a shorter effective green time for a lane with opposed turns compared with adjacent through traffic lanes. This method cannot be used for shared lanes unless a lane-by-lane analysis method is adopted. It gives more accurate capacity and performance prediction compared with the adjustment factor method which is associated with the method of analysing by lane groups (8,9).

Using the lost time method, explicit modelling of protected-permitted turns, slip lanes, turns on red, etc. is achieved in a general way for exclusive and shared lanes without need for the use of complicated adjustment factor equations.

The opposed turn model in SIDRA takes into account the number of lanes and the individual lane characteristics of opposing movements (there can be several of them) in estimating the blocked interval lengths and filter turn saturation flow rates. The parameters of the gap acceptance model used for this purpose (critical gap, follow-up headway, departures after the end of green period) can be specified for individual movements. The user can also specify different priority rules for opposed and opposing turns. For opposed turns from shared lanes, a variable number of vehicles turning at the end of green time is allowed as in the HCM, but the user can specify the maximum number of vehicles that can depart. A detailed discussion of the subject can be found in references (8-11).

2.5 Lane Blockage in Shared Lanes

In shared lanes, *lane blockage* is modelled explicitly and in a very general way that allows for any type of interaction between movements in the lane. The model predicts capacities due to departures before being blocked using *free queues* and lane flows as variables, and treats the intervals of no departure (i.e. when the lane is blocked) as *lost time* (or effective red). This model is useful for all cases of opposed turns in shared lanes including the complicated cases of protected-permitted turns, slip lanes and turns on red.

2.6 Right Turn on Red

The HCM/HCS methods for estimating the effects of right turns on red (RTOR) is to subtract the RTOR volume from the total right-turn volume. This method is inadequate since the RTOR volume should vary depending on intersection geometry, volumes and signal timings. At the same time, the volume subtraction method is never a satisfactory method for performance estimation (full flow rates are needed for predicting lane blockage affects, short lane capacities, queue lengths, delays, etc.).

SIDRA offers a comprehensive method of modelling RTOR (or left turn on red for driving on the left), which makes use of the general opposed turn (protected-permitted) and shared lane modelling features employed in SIDRA (14). The method is similar to the gap acceptance based method described in a recent paper (37), but it is a fully developed capacity and performance prediction method. The same method is also applicable to traffic in *slip lanes.*

2.7 Short Lanes

Capacities of *short lanes* in the approach road (turn bays, or lanes with parking) are modelled in a detailed way. *Excess flows* from short lanes are added to adjacent lane

flows when the queue storage capacities are exceeded. Short lanes are an important aspect of signalised intersection operation which has been neglected in the HCM/HCS and the OSCADY program, while the Canadian and the Swedish methods allow for short lanes (38,39).

Downstream short lanes (merging cases) are taken into account through reduced utilisation of the corresponding approach (upstream) lane.

2.8 Saturation Flow Adjustment Factors

SIDRA accounts for the effect of *turn radius* on saturation flow explicitly. In addition to the HCM method of adjusting saturation flows, the effect of conflicting pedestrian streams on turning traffic can be accounted for by using the lost time (effective red time) method. In modelling this type of effect on saturation flows, including opposed turn modelling, there is no limitation in terms of left or right turns, i.e. the models are generally applicable to all types of turns.

Variable passenger car equivalents are used for turning heavy vehicles as a function of the turn radius and conflicting pedestrian volume. The concept of excess headway equivalent has been introduced for this purpose (11).

2.9 Volume Adjustment

In modelling unequal lane utilisation, protected-permitted turns, slip lanes, turns on red, etc., SIDRA does not adjust or manipulate arrival flow rates. Arrival flows are always used in *vehicle units*, and are only adjusted for peak hour factors and flow scales (growth factors). Arrival flows are never reduced in lieu of capacity effects (e.g. for RTOR effects in the HCS). All capacity effects are modelled by adjusting saturation flows and effective green times. This achieves a consistent method in capacity and performance prediction as discussed in ARR No. 123 (1).

Thus, the SIDRA method is in agreement with a suggestion to modify the HCM volume adjustment method for lane underutilisation (33). However, the suggestion that peak hour factors should also be applied as saturation flow adjustments rather than demand volume adjustments (33) is not agreed with. This issue needs to be discussed in relation to variable demand modelling (45).

2.10 Heavy Vehicle Data

Heavy vehicle volumes can be specified as percentage or actual vehicle values. Different heavy vehicle percentages can be specified for individual movements (lane groups). Different values of queue space per vehicle are used for light and heavy vehicles in order to determine queue lengths. These values also affect short lane capacities which depend on the number of available queue spaces. SIDRA calculates queue lengths for individual lanes according to the traffic mix in the lane.

2.11 Performance Measures

The performance measures predicted by SIDRA include not only delay and queue length but also such characteristics as the number of stops, fuel consumption, pollutant emissions and operating costs. Fuel consumption is estimated using a four-mode elemental model which is based on ARRB work that won an ITE award in 1986 (42,43). The parameters of this model can be specified for local vehicle and driving characteristics (11).

In SIDRA, delay can be defined as *stopped* or *overall* delay. Levels of service are determined accordingly (11,14). The HCM progression factors for delay calculations are applied on a lane by lane basis. Since individual lanes in a lane group can have different degrees of saturation, and a shared lane can have turns with different arrival types, SIDRA calculates flow weighted average progression factors for movements (lane groups) which may differ from the corresponding HCS results. SIDRA allows the user to specify signal and arrival types for individual movements (lane groups), and uses an *uncoordinated turn* type. These features help to overcome various limitations of the HCM method (33).

SIDRA provides a generalised delay formula which can be calibrated through the Default Values File (7,11). An important issue in the calibrated and application of delay equations is the use of the delay formula for individual lanes (as in SIDRA and the Swedish models) rather than lane groups (as in the HCM/HCS and the OSCADY program). In multi-lane cases, smaller delays are obtained with the application of the delay formula on a lane group basis as discussed previously (2).

For example, consider arrival and saturation flow rates of 800 and 1600 veh/h per lane for one, two and four lane cases with identical conditions for individual lanes ($y=0.5$). For cycle time = 100 s, effective green time = 50 s (hence, $x = 1.0$) and progression factor = 1.0, SIDRA HCM version predicts an average delay of 43.7 s. On the other hand the HCS package will predict decreasing delays with increasing number of lanes in the group (43.5, 36.3 and 31.2 s, respectively). This may lead to misleading results in evaluating signal design options since higher delays may be predicted when lane arrangements are changed from shared lanes to exclusive lanes, e.g. from (LT,T), which is a lane group with two lanes, to (L,T), which is two lane groups with one lane each (L = Left, T = Through).

The lane-by-lane application of the delay formula will also give better results in the case of unequal lane utilisation. These comments apply to all performance parameters (delay, queue length, stop rate, etc.)

2.12 Level-of-Service Options

SIDRA offers the following Level-of-Service definitions as options (11,14):

(a) *Delay* only (stopped delay as defined by the HCM, or overall delay including stop-start delays);

(b) Both *delay* and the *degree of saturation* (v/c ratio) as specified by Berry (20);

(c) The *degree of saturation* only when delay calculations are not carried out (a SIDRA option); this could be useful for planning analysis purposes, e.g. see Cassidy and May (27);

(d) The *density* for uninterrupted movements (as specified in Chapters 3 and 7 of the HCM, but using passenger car *space* equivalents).

2.13 Movement Data

In SIDRA, different ideal saturation flows, free queues (for lane blockage), speeds, queue spaces per vehicle, progression factors (signal and arrival types), etc. can be specified for individual movements. These different movement parameters are taken into account in shared lane capacity and performance predictions. The facility for *mixed flows* (a more advanced form of the TRANSYT shared stop-line facility) enhances this process.

2.14 Critical Movement Analysis

SIDRA employs a very general *critical movement identification* method as a basis for computing *signal timings* (cycle time and green splits) for simple as well as complex phasing arrangements. The basic principles of the method have been described in ARR NO. 123 (1),and the method has been improved in SIDRA to handle the case of two green periods per cycle.

Unlike the HCM critical analysis method, the SIDRA method takes right turns into account. Any combination of overlap movements and two separate green periods per cycle for any movement are allowed. Different minimum and maximum green time constraints and different practical (maximum acceptable) degrees of saturation can be specified for different movements. Selected movements and individual green periods can be specified as *Undetected* in order to eliminate them from the critical movement analysis.

The limitations of the HCM critical movement analysis method as specified for the planning analysis procedure (24,27,34,35) include:

(a) neglecting right turn movements, e.g. Eastbound right-turn lane should be critical in the HCM Example 4 (p.9-57) revised geometry case;

(b) neglecting minimum (and any maximum) green times;

(c) independence from cycle time (SIDRA uses *required times* in critical movement analysis which indicates that critical move ments may change depending on the cycle time, especially because of minimum green time requirements);

(d) *implying* a specific phasing arrangement (as a result, it is not generally applicable, e.g. it does not apply to a simple two-phase arrangement with permitted turns).

2.15 Signal Timings

SIDRA can be used to determine both the cycle time and green splits, or only the green splits for a given cycle time. Alternatively, both cycle time and green splits can be given for capacity and performance estimation only. The user can specify *green split priorities* for selected movements, for example, for major road traffic (11,44). *Undetected* movement and *phase* deletion facilities are also available.

2.16 Variable Cycle Time

A *variable cycle time* facility allows the user to establish the relationship between the cycle time and capacity and other performance measures. An interesting feature of SIDRA is to indicate that the capacity can decrease as a result of increased cycle time. An example is given Figure 1 (see reference (12) for details). GOSID program display graphs of delay, queue length, capacity, etc. for variable cycle time runs.

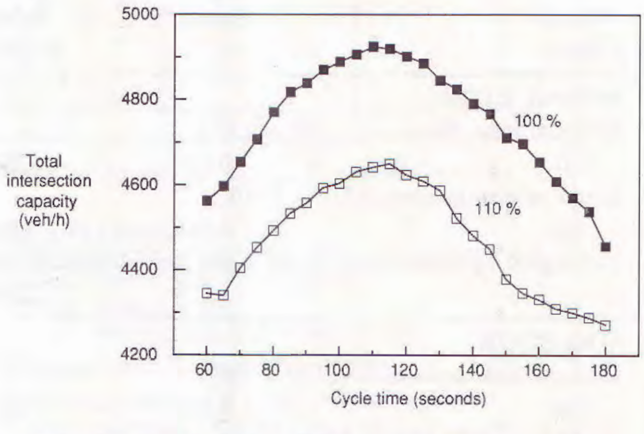


FIGURE 1: Example Showing a Decrease in Total Intersection Capacity with Increased Cycle Time (for flow scales of 100 and 110 per cent)

2.17 Variable Flow Scale

Sensitivity analyses and intersection design life calculations can be carried out using the *variable flow scale* facility of SIDRA. This allows the user to change demand flow levels for all movements or for a selected

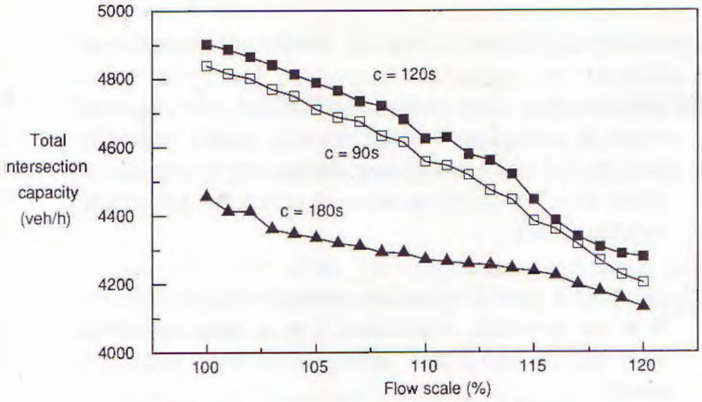


FIGURE 2: Example Showing a Decrease in Total Intersection Capacity with Increased Demand Flow Level (for cycle times of 90, 120 and 180 s)

group of movements. SIDRA results indicate that increased demand flow levels often result in decreased capacities. An example is given in Figure 2 (see reference (12) for details). GOSID program displays graphs of delay, spare capacity, etc. for variable flow scale runs.

2.18 Iterative Method

SIDRA employs an *iterative approximation* method of computation. *Main iterations* are carried out to allow for the interdependence of capacity and timing parameters, and *sub-iterations* are carried out to allow for the interdependence of capacities and lane flows of approach roads using the same signal phase (particularly with opposed turns in shared lanes).

3. EXAMPLE

A SIDRA Input Data Preparation Form for the HCM Example 3 (p. 9-50), and selected tables from SIDRA output (run with the timings calculated in the HCM) are given in Figures 3 and 4. Although SIDRA and HCS gave close results for the intersection degree of saturation, delay and level of service, substantial differences were found in left turn capacity and delay predictions. Problems of the HCM/HCS method with capacity and delay prediction for permitted left turns in this example were discussed previously (5,6); also see Bonneson and McCoy (21). Signal timing results from SIDRA were close to those calculated in the HCM (c = 120 s with up to 5 s difference in green times), the main difference being mainly due to the deletion of phase B by SIDRA.

To demonstrate the effects of RTOR and lane utilisation factors, the SIDRA results with and without RTOR, and with equal and unequal lane utilisation (LUF = 1.00 and 1.05) are given in Table 2. It is seen that the assumption about unequal lane utilisation implies substantial deterioration of intersection performance. The delay benefits from RTOR for this example (which has small

TABLE 2: Intersection Delay and Los Results for the HCM Example 3: Effects of Lane Utilisation and Right Turn On Red (RTOR) *

Cycle Time c (sec)	Deg. of Satur. x	Aver. Delay d (sec)	Level of Service	Longest Queue (vehs)	Fuel Eff. (mi/ga)
Without RTOR					
Unequal lane utilisation (LUF = 1.05)					
120	0.958	25.9	D	34.0	19.4
Equal lane utilisation (LUF = 1.00)					
86	0.940	20.3	C	23.4	19.9
Difference between unequal and equal lane utilisation cases					
	2 %	22 %		31%	3 %
With RTOR					
Unequal lane utilisation (LUF = 1.05)					
120	0.947	24.6	C	32.6	19.6
Equal lane utilisation (LUF = 1.00)					
87	0.930	19.0	C	22.9	20.1
Difference between unequal and equal lane utilisation cases					
	2 %	23 %		30 %	3 %
RTOR Benefits					
Unequal lane utilisation (LUF = 1.05)					
	1 %	5 %	=	4 %	1 %
Equal lane utilisation (LUF = 1.00)					
	1 %	6 %	–	2 %	1 %

* Based on SIDRA timings (single run; 1-sec. cycle increment; maximum cycle time = 120 s)

= LOS changes from D to C, but the delay change is marginal.

SIDRA INPUT DATA PREPARATION FORM

(FOR RIGHT–HAND DRIVING)

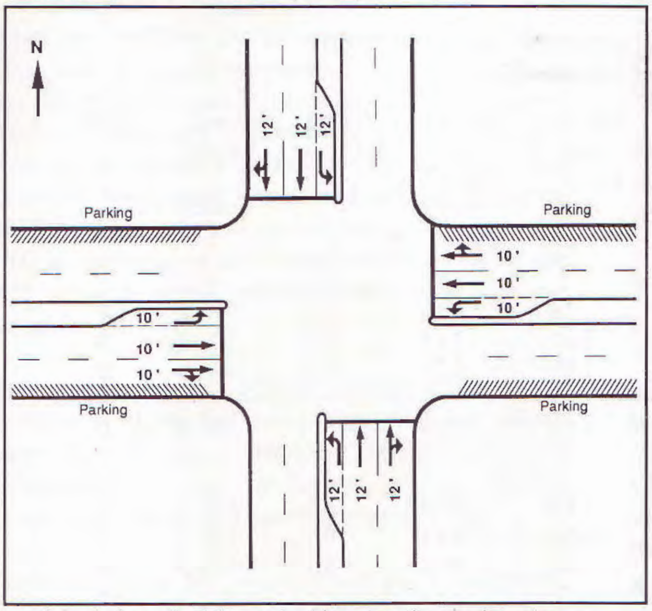
Prepared by : *Rahmi Akcelik* Computer File Name : *US3A*

Date : *April 1990* Reference No. :

Intersection Title : *HCM Example 3 (p. 9-50)*

Run Description : *Short lanes neglected; Unequal lane utilisation*

INTERSECTION LAYOUT (Description:)

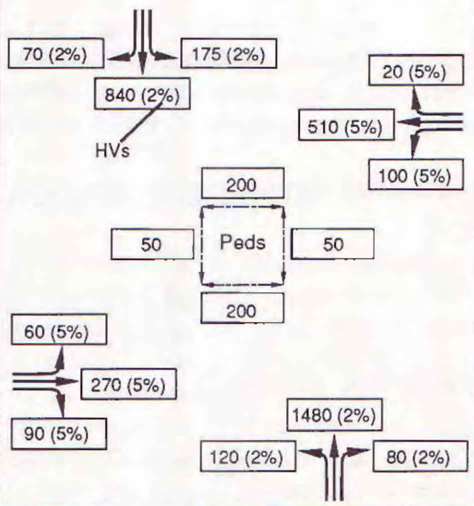


Include lane disciplines, short lane lengths, grades, etc.
Enter a description such as existing or proposed.

VOLUMES (Unit time: *60 min*)

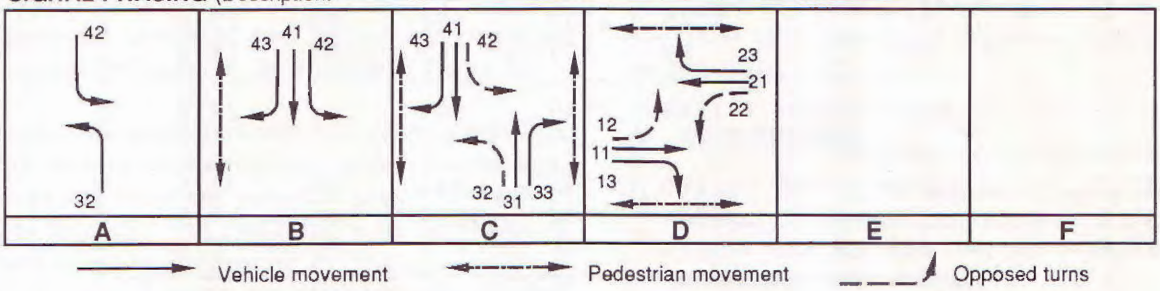
Heavy vehicle counting method: *P*

P = Percent S = Separate T = included in Total

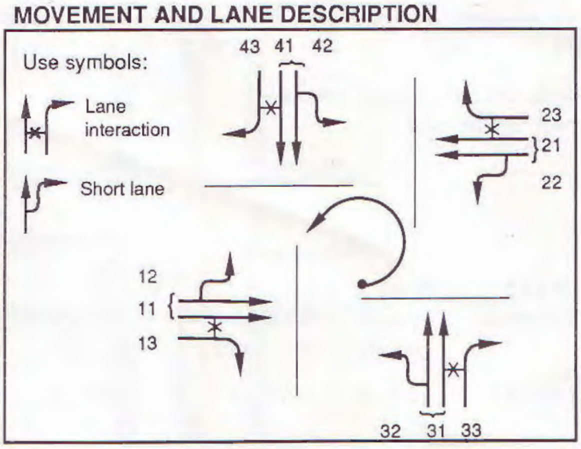


Show pedestrian volumes if necessary.

SIGNAL PHASING (Description:)



MOVEMENT AND LANE DESCRIPTION



OTHER FEATURES

- *Area type: CBD (Env. code=2)*
- *Peak Hour Factors:*
  E-W: PHF=0.85, N-S: PHF=0.90
- *Signal type: Actuated (x_p=0.95)*
- *Arrival type: 3 (PF=0.85) for all movs*
- *Lane underutil.: LUF=1.05 for (T+R) lanes*
- *Parking: E-W: N_p=5. All grades=0*
- *All movements: I=l=3 s (unopposed)*
- *Mov. 42: start loss in Phase C=0*
- *Mov. 12: end departures n_{fm}=2.5*
- *G_{min}=22 s (pedestrians)*
- *HCM timings: c=119 s, G_i=5,2,71,29 s*

Specify movement types, turn types, timing data, etc.

FIGURE 3: SIDRA Input Data Preparation Form for the HCM Example 3

A R R B - S I D R A 3.2

Australian Road Research Board
Rahmi Akcelik Registered User No. 1
Time and Date of Analysis 17:04:02 1990-04-30

HCM EXAMPLE 3 (P.9-50) RTs and LTs as SEPARATE Movs .... * US3A *
Short lanes neglected; Unequal lane util. --- TIMINGS GIVEN

SIDRA US Highway Capacity Manual (1985) Version

Cycle Time = 119

Table S.15 - CAPACITY AND LEVEL OF SERVICE (HCM METHOD)

Mov No.	Mov Typ	Green Time Ratio (g/C) 1st grn	2nd grn	Total Flow (veh /h)	Total Cap. (veh /h)	Deg. of Satn (v/c)	Prog. Factor	Aver. Delay (sec)	LOS
EASTBOUND APPROACH									
12	L	.025		71	76	.939	.85	88.2	F
11	T	.244		318	447	.712	.85	30.4	D
13	R	.244		106	165	.641	.85	29.4	D
		Mov. Group:		495	688	.939		38.5	D
WESTBOUND APPROACH									
22	L	.092		118	119	.989*	.85	86.1	F
21	T	.244*		600	612	.981	.85	50.7	E
23	R	.244		23	26	.882	.85	42.9	E
		Mov. Group:		741	757	.989		56.1	E
NORTHBOUND APPROACH									
32	L	.042	.462	134	350	.383	.85	6.0	B
31	T	.597*		1645	1707	.964	.85	23.8	C
33	R	.597		89	103	.867	.85	18.4	C
		Mov. Group:		1868	2159	.964		22.2	C
SOUTHBOUND APPROACH									
42	L	.084*	.084	195	231	.844	.85	33.5	D
41	T	.639		934	1768	.528	.85	7.8	B
43	R	.639		78	164	.475	.85	7.5	B
		Mov. Group:		1207	2163	.844		11.9	B
		Intersection:		4311	5971	.989		27.0	D

Level of Service calculations are based on stopped delay.
* Maximum v/c ratio, or critical green periods

Table S.6 - INTERSECTION PERFORMANCE

Total Flow (veh/h)	Total Delay (veh-h/h)	Aver. Delay (sec)	Total Stops (veh/h)	Stop Rate	Perf. Index	Aver. Speed (mi/h)	F U E L Eff. (mi/ga)	Total (ga/h)
4311	32.39	27.0	3757	.87	248.81	23.6	19.2	127.2

Progression Factors apply to delays only.
Queue length and stops are based on random arrivals (no platooning).

FIGURE 4: SIDRA Output Tables S.15 and S.6 (summary form) for the Example in Fig. 3

right-turn volumes in shared lanes) are of the order of 5 per cent.

For detailed discussion of the application of SIDRA to Examples 1 to 5 of the signalised intersection chapter of the HCM, the reader is referred to SIDRA document DN 1709 (14).

4. CONCLUSION

This paper has described many features of SIDRA which could be used to overcome various limitations of the current HCM/HCS methodology for signalised intersections. However, it should be pointed out that SIDRA is by no means a perfect model. Since its first release, it has been under continuous revision either by developing new models and techniques through research at ARRB in response to user feedback, or by using the results of research carried out elsewhere (including some features of the HCM).

The most recent version of SIDRA incorporates graphics-based input and output programs. **RIDES** (**R**oad **I**ntersection **D**ata **E**diting **S**ystem) and **GOSID** (**G**raphical **O**utput **S**ystem for **I**ntersection **D**esign), and the **VIEWS** (On-screen Text **VIEW**ing **S**ystem) program (13).

With extensive picture facilities to check and present data, intersection geometry and phasing data specification in design style, automatic/interactive data specification features, and an input data file library facility, RIDES makes input data preparation an easy task for both new and experienced users alike (see Figure 5 for the picture of intersection geometry for the example in Figure 3).

GOSID provides output in picture form displaying SIDRA capacity and performance results for movements, approaches and lanes, and generates graphs for variable cycle and flow scale runs (see Figure 6 for the display of movement delays for the example in Figure 3). This helps the user in understanding the SIDRA results. VIEWS allows the user to inspect the detailed SIDRA output (text files) without need for a text editor.

SIDRA can be used for capacity and performance analysis of roundabouts and other unsignalised (give-way and stop-sign controlled) intersections. It uses a roundabout model based on research carried out at the Australian Road Research Board (46,47). The capability to analyse roundabouts and other unsignalised intersections in addition to signalised intersections makes SIDRA a comprehensive intersection analysis tool that uses consistent methodology and saves input data preparation time.

Research work on modelling platooned arrivals and queue interactions in closely-spaced and large intersections is in progress at ARRB (Paired Intersections Project).

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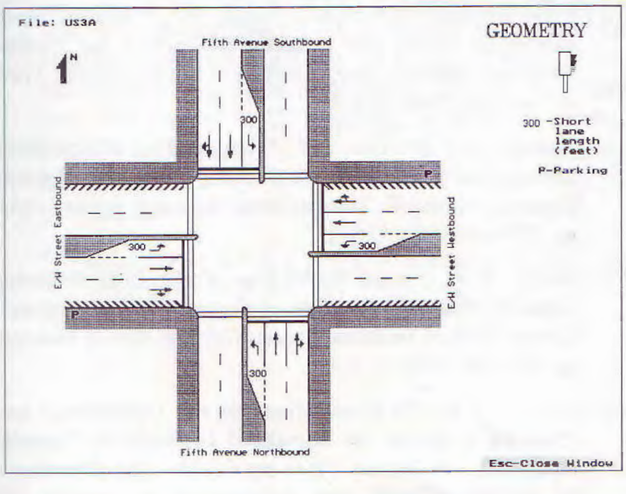

FIGURE 5: The intersection Geometry Picture Generated by RIDES for the Example in Figure 3

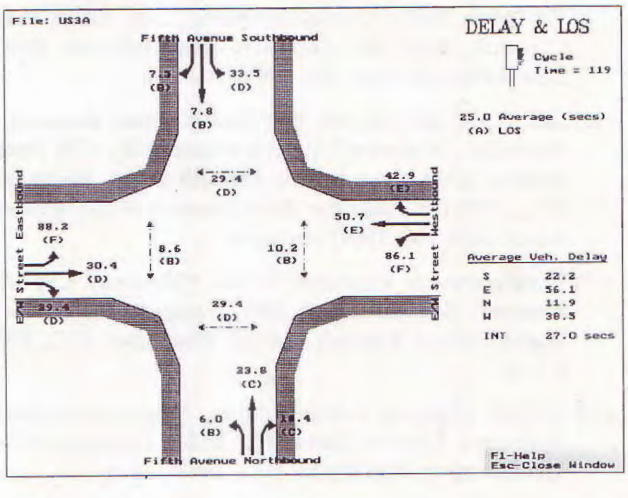

FIGURE 6: Display of Movement Delays and Levels of Service for the Example in Figure 3 (GOSID screen)

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